PECAN CREEK, GAINESVILLE, TEXAS
DETAILED PROJECT REPORT
AND
INTEGRATED ENVIRONMENTAL ASSESSMENT

APPENDIX C.1
HYDROLOGY AND HYDRAULICS

Basin Description

The City of Gainesville, county seat of Cooke County, Texas, is located adjacent to Interstate Highway 35 approximately 65 miles north of the Dallas-Fort Worth metropolitan area and 7 miles south of the Oklahoma state line.

The Pecan Creek watershed, which abuts Red River watershed approximately 6 miles north of Gainesville, is situated entirely within Cooke County. Pecan Creek flows southward through the central portion of the city, eventually reaching its confluence with the Elm Fork of the Trinity River, approximately 3 miles south of the city. This study focuses on the portion of the Pecan Creek watershed situated upstream from the confluence of one of its major left bank tributaries, Wheeler Creek. The contributing drainage area of Pecan Creek is approximately 12.4 square miles above the city and 15.5 square miles above the confluence of Wheeler Creek, which is located approximately 2 miles south of the city. This portion of the Pecan Creek watershed is generally triangular in shape, with a total length of approximately 8.0 miles. The average watershed width is approximately 2.0 miles and the maximum watershed width is approximately 5.5 miles. Topographic elevations within this portion of the Pecan Creek watershed range from a maximum of 965 feet to a minimum 687 feet National Geodetic Vertical Datum (NGVD).   A detailed watershed map is provided as Figure-1.

The majority of the Pecan Creek watershed, situated both upstream and downstream from the City of Gainesville, is utilized for agricultural production purposes including both farming and ranching activities. The non-cultivated portion of these lands are predominantly grass-covered with terrestrial zones generally limited to relatively narrow bands along a few, well-defined water courses. The portion of the watershed situated within the City of Gainesville exhibits a general mix of residential, commercial, and light-industrial uses. Additionally, the Pecan Creek valley serves as the corridor for the Atchison, Topeka and Sante Fe Railway, a major transporter of goods through North Texas.

The Natural Resources Conservation Service (NRCS) "*Soil Survey of Cooke County, Texas*", identifies this study area watershed as having surface soils comprised primarily of the Normangee-Wilson-Crockett soil association, characterized as "loamy soils that are deep, nearly level to sloping, on uplands and terraces". A closer inspection of the published physical and chemical properties table reveals that the predominant soil types in this area are comprised of a shallow (typically 7-inch) layer of sandy soil atop a deep (typically 5-foot)

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MAP 1 – Subbasin Delineation
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layer of clayey soils, exhibiting permeability rates of less than 0.06 inches per hour. In contrast, however, narrow strips of land along the well-defined watercourses are identified as being comprised of deep (typically 5-foot) layers of sandy soils, exhibiting permeability rates of 0.6 to 2.0 inches per hour.

Flood History

The major storms of the area are generally produced by frontal-type activity occurring mainly in the spring and fall but occasionally at other times of the year. These storms are usually intense localized thunderstorms resulting in significant rainfall amounts.

October 1981 - Gainesville recorded a total rainfall of 23.55 inches for the period 6-14 October 1981 with 6.9 and 7.25 inches falling on 12 and 13 October, respectively. Resultant flooding damaged properties along Pecan Creek and the Elm Fork of the Trinity River. Based on regional rainfall statistics, a storm of this magnitude has a 2 percent exceedance probability.

May 1989 – A storm event occurred on May 16, 1989 in the Gainesville area that produced major flooding on Pecan Creek. Hourly rainfall records available from the National Weather Service show that 3.9 inches of rainfall occurred in the one-hour period of 10:00 am to 11:00 am on May 16, 1989. Over the 3-hour period of 10:00 am to 1:00 pm, rainfall totaled 5.8 inches. Most of the rainfall for this storm was concentrated on the Pecan Creek and Wheeler Creek watersheds, resulting in wide spread flooding of homes and businesses along these two streams. Based on regional rainfall statistics, a storm of this magnitude has a 1 percent exceedance probability.

April 1990 – A storm event occurred on April 25, 1990 that produced 5.35 inches of rainfall in a 21-hour period in Gainesville. Minor flooding resulted on Pecan Creek. Based on regional rainfall statistics, a storm of this magnitude has a 20 percent exceedance probability.

May 1993 – A storm event occurred on May 9, 1993 that produced approximately 3.9 inches of rainfall in a 4-hour period at Gainesville. The storm resulted in flooding of low-lying areas adjacent to Pecan Creek. Based on regional rainfall statistics, a storm of this magnitude has a 20 percent exceedance probability.

Prior Studies

The currently effective Flood Insurance Study (FIS) for the City of Gainesville is dated 15 April 1981. Hydrologic analysis methods employed by the FIS study contractor (Bovay Engineers, Inc.) were based on application of regional runoff equations developed by the US Geological Survey (USGS) in 1977. The portion of their hydrologic analysis results relating to the current study, as tabulated in the FIS report, is listed below in Table 1.

Table 1 - FIS Discharges in cubic feet per second (cfs)

PECAN CREEK LOCATION	DRAINAGE AREA (sq.miles)	ANNUAL EXCEEDANCE PROBABILTY (percent)			
		10	2	1	0.2
		RECURRENCE INTERVAL (years)			
		10	50	100	500
At 2,000 feet upstream from IH-35	1.9	1,270	2,140	2,540	3,650
Upstream from confluence of North Tributary	3.4	1,810	3,060	3,650	5,280
At 2,000 feet downstream from US Highway 82	13.5	4,210	7,420	8,970	13,400
Upstream from confluence with Wheeler Creek	15.3	5,3100	8,690	10,300	14,800

A Section 205 Detailed Project Report (DPR) for Pecan Creek at Gainesville was published in August 1986. This particular study, developed by the Fort Worth District US Army Corps of Engineers, was similar in scope to and based on similar methodologies as those applied in the current study. The hydrologic analysis for existing conditions was completed in 1983. The portion of the hydrologic analysis results relating to the current study, as tabulated in the DPR report, are shown in Table 2.

Table 2 - Discharges in Cubic feet per second from 1986 Corps Feasibility Report

LOCATION DESCRIPTION	CONTRIBUTING WATERSHED AREA (sq.miles)	ANNUAL EXCEEDANCE PROBABILTY (percent)						
		50	20	10	4	2	1	0.2
		RECURRENCE INTERVAL (years)						
		2	5	10	25	50	100	500
At IH-35	2.82	1486	2405	2909	3520	4019	4535	5668
Above Confl. With Trib Pec-3	3.54	1493	2525	3039	3667	4203	4755	6019
Below Confl. With Trib Pec-3	10.43	2256	4026	5382	7003	8360	9704	12969
Above Confl. With Trib Pec-2	10.82	2211	3979	5328	6872	8297	9642	12932
Below Confl. With Trib Pec-2	12.64	2723	5115	6432	7909	9509	11218	15068
At Belcher Street	14.31	2611	4904	6384	8001	9672	11421	15482
At California Street	14.70	2578	4825	6322	7948	9617	11364	15502
At Garnett Street	14.86	2569	4809	6307	7935	9504	11220	15519
At Anthony Street	15.63	2538	4739	6241	7872	9479	11212	15597
Above Confl. With Wheeler Cr.	16.06	2501	4688	6185	7814	9442	11162	15589

Henningson, Durham, and Richardson, Inc published a Flood Protection Planning Study for the City of Gainesville in May 1999. Their study involved investigation of a wide range of flood damage reduction alternatives on Pecan Creek and several other flooding sources around the community. Their hydrologic analysis methodology was based on

application of the SCS (Soil Conservation Service) Method, which required determination of runoff curve numbers for each subbasin and dimensionless unit hydrographs. The portion of their hydrologic analysis results relating to the current study, as tabulated in their report, is summarized in Table 3.

Table 3 - Summary of Discharges from Henningson, Durham, and Richardson, Inc Report

LOCATION	CONTRIBUTING WATERSHED AREA (sq.miles)	ANNUAL EXCEEDANCE PROBABILITY (percent)						
		50	20	10	4	2	1	0.2
		RECURRENCE INTERVAL (years)						
		2	5	10	25	50	100	500
IH-35	3.0			1720	2140		2820	
Highway 82	12.4			4630	5930		8220	
Belcher Street	14.0			5360	6330		8890	
California Street	14.5			4440	5980		8300	
Anthony Street	15.0			4340	5870		7960	
Upstream Of Wheeler Creek	15.4			4190	5510		8050	

Climatology of the Area

The Pecan Creek watershed is located in a region of temperate mean climatological conditions, experiencing occasional extremes of temperature and rainfall of relatively short duration. Tropical air masses from the Gulf of Mexico play a dominant role in the climate of the area during spring, summer, and fall. Modified polar air masses (cold fronts) contribute significantly to the winter climate with sharp drops in the temperature and strong, gusty northerly winds. These cold fronts move through rapidly, and periods of fair, mild weather occur often. The National Oceanic and Atmospheric Administration (NOAA) at Gainesville has recorded a maximum temperature of 114 degrees Fahrenheit occurring on 10 August 1936, and a minimum temperature of –12 degrees Fahrenheit occurring on 12 February 1899.

The precipitation gaging records at Gainesville have been almost fully continuous since 1897. The mean annual rainfall is approximately 35 inches, while the annual maximum and minimum rainfall are 69.66 and 16.19 inches occurring in 1900 and 1963, respectively. Rainfall is generally uniformly distributed throughout the year except for typically dry periods, occurring between December and February. Thunderstorms of short duration may occur at any time during the year, while snowfall occurs so rarely as to have little hydrologic significance.

Streamflow Gaging

There are no streamflow gaging stations in the immediate study area.   However, there is a recording rain gage in the city of Gainesville.   In addition, two small watersheds in the vicinity, NRCS Subwatershed Number 6 near Muenster and Little Elm Creek NRCS Subwatershed No. 10 near Gunter, are equipped with both rainfall and discharge recording devices. Storms occurring on these watersheds were considered in the derivation of rainfall losses (both initial abstractions and infiltration rates) and unit hydrograph coefficients for the Pecan Creek watershed runoff model.

Development of Discharge - Frequency Relationships

The existing HEC-1 watershed runoff model, prepared by HDR, Inc. for the City of Gainesville, was used as a baseline model for the current study. The various input parameters were either verified or updated, as necessary for compatibility with standard modeling methodologies used on preceding Section 205 Flood Damage Reduction Feasibility Studies by the Fort Worth District, US Army Corps of Engineers. Based upon the relatively slow growth of urbanization upstream from the City of Gainesville and the fact that the concurrent flood damage reduction economics analyses aim to provide present-value benefits for merely a 50-year future, it was not deemed necessary to configure the modeling for any future urbanization condition during the planning phase of this potential project.

The Pecan Creek watershed was subdivided into runoff subbasins in an identical fashion as performed by HDR, Inc. Watershed and subbasin divides were carefully checked and adjusted where necessary. This involved a close inspection of 2-foot contour interval detailed topographic mapping provided by the City and the standard 7.5-minute, 10-foot contour interval, USGS topographic mapping quadrangles entitled: "Gainesville North", "Callisburg", "Woodbine", and "Gainesville South". There are a total of 21 runoff subbasins configured for the portion of the Pecan Creek watershed above the confluence of Wheeler Creek. These subbasins range in drainage area size from 0.25 to 1.44 square miles.   A detailed watershed map is provided as Figure 1. Directly measurable subbasin parameters (Area, Length, Length-to-Centroid, and Weighted Stream Slope) were computed electronically, via Bentley MicroStation software.

Development of Rainfall-Frequency Relationships

Hypothetical point rainfall depths for the 50-, 20-, 10-, 4-, 2-, 1-, 0.4-, and 0.2-annual percentage chance exceedance (2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year) storms was developed by splicing data from the National Weather Service (NWS) Technical Paper Number 40 (TP 40) and the NOAA Technical Memorandum Hydro-35. Rainfall for the 500-year frequency storm was computed by extrapolation of this data. Table 4summarizes the hypothetical point rainfalls.

Table 4 - Hypothetical Rainfall Data (inches)

	ANNUAL EXCEEDANCE PROBABILITY (percent)							
	50	**20**	**10**	**4**	**2**	**1**	**0.4**	**0.2**
	RECURRENCE INTERVAL (years)							
	2	**5**	**10**	**25**	**50**	**100**	**250**	**500**
STORM DURATION								
5 minutes	0.49	0.57	0.63	0.73	0.80	0.87	1.05	1.15
15 minutes	1.02	1.21	1.35	1.55	1.71	1.87	2.17	2.35
1 hour	1.80	2.31	2.64	3.13	3.51	3.88	4.42	4.80
2 hours	2.15	2.91	3.45	4.00	4.55	5.00	5.75	6.30
3 hours	2.39	3.20	3.80	4.45	5.04	5.65	6.45	7.05
6 hours	2.84	3.85	4.55	5.35	6.05	6.80	7.78	8.55
12 hours	3.36	4.55	5.35	6.28	7.15	8.00	9.10	10.00
24 hours	3.90	5.26	6.22	7.32	8.28	9.30	10.57	11.65

The representative rainfall depths for these frequency-related storms were transposed to provide appropriate average magnitudes over the specific contributing drainage area size at each point of hydrologic interest, based upon the family of curves within Figure 15 of TP 40. A mass rainfall curve was generated from the adjusted point rainfall values for each frequency, interpolated into incremental amounts, and distributed at 5 minute time intervals. Figure 1 presents a sample of the 1-percent annual chance exceedance (100-year) storm applied over a 1.40-square mile subbasin.

Figure 1 - Sampe Rainfall Pattern
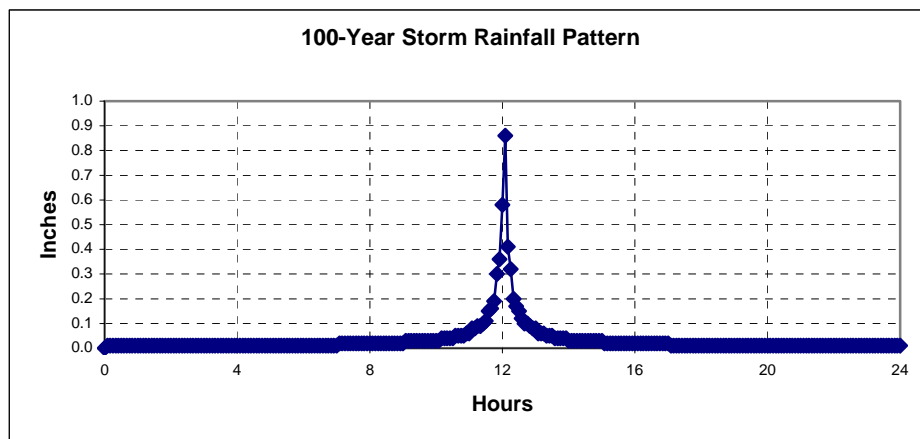
Determination of Percent Sand

Standardized Fort Worth District US Army Corps of Engineers methodologies utilize the *percent- sand* parameter as an indices for estimation of initial abstractions and infiltration rates. The computation of each subbasin's soil type was determined by comparing the permeabilities of the major soil types in this watershed with those found in the Blackland Prairie and Cross Timber regions.

The Blackland Prairie is a nearly treeless, mostly flat grassland, with the majority of the unurbanized areas under cultivation. Soils typical of the Blackland Prairie are clays and clay loams, such as the Houston Black soil series. This series consists of moderately well-drained, deep, cyclic, clayey soils, which formed in alkaline, marine clay, and material weathered from shale. It is typically found on lands sloping from 1 to 4 percent, and has a permeability of less than 0.06 inches per hour. It is the predominant series found in watersheds used to develop the Blackland Prairie Clay Urbanization Curves, Figure 2 (discussed later in this narrative). Soils having permeabilities of less than 0.06 inches per hour have been assigned a percent sand value of 0, for use with the urbanization curves.

Figure 2 - Blackland Prairie Clay Urbanization Curves
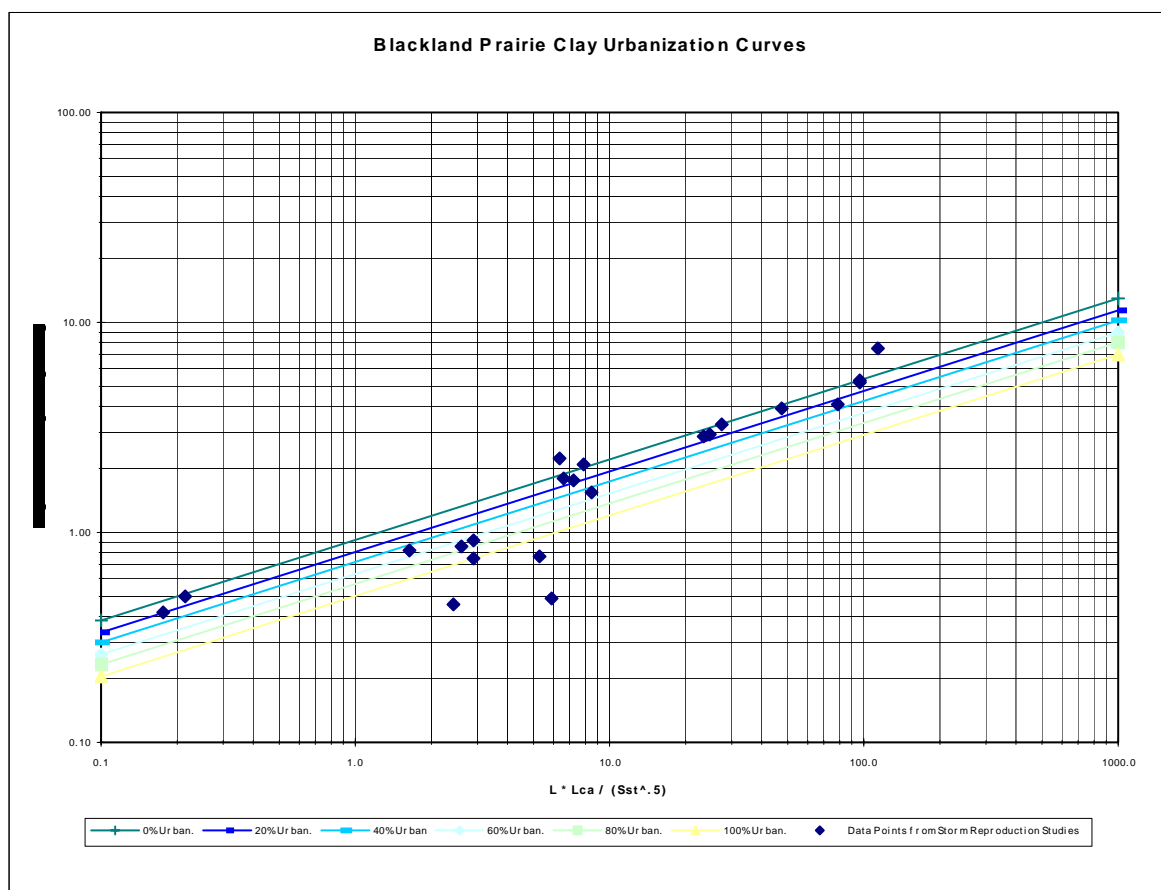
The Cross Timber is a wooded region adjacent to the Blackland and Grand Prairies. Soils typical of the Cross Timber are fine sandy loams, such as the Crosstell soil series.   This series consists of moderately well-drained, deep, loamy soils on uplands that formed in shaley and clayey sediment containing thin strata of weakly cemented sandstone. It is typically found on lands sloping from 1 to 5 percent, and has a permeability between 0.6 and 2 inches per hour. It is the predominant series found in the watersheds used to develop the Cross Timber Sandy Loam Urbanization Curves, Figure 3, discussed later in this narrative. Soils having permeabilities between 0.6 and 2 inches per hour have been assigned a percent sand value of 100, for use with the urbanization curves.

Figure 3 - Cross Timber Sandy Loam Urbanization Curves
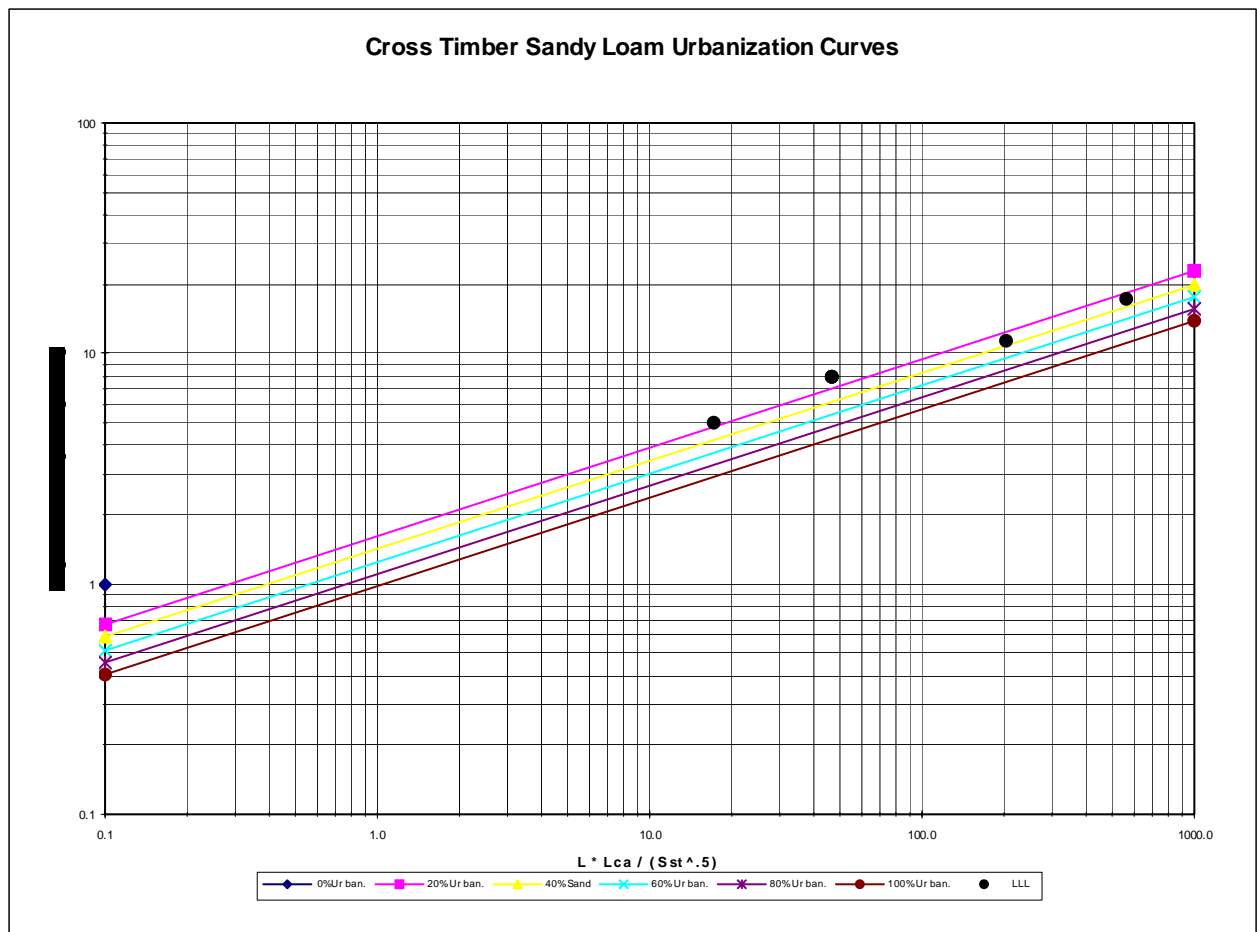
Soils with permeabilities ranging between 2 and 6 inches per hour were assigned a 133 percent sand value and the few with permeabilities greater than 6 inches per hour were assigned a 167 percent sand value, for use with the urbanization curves. Percent sand values for soil types with permeabilities between those for clayey and sandy soils were linearly interpolated. The appropriate percent sand value for each soil type was multiplied by the percent of the subbasin covered by that soil type, and the product summed for all soil types within each subbasin, in order to develop an overall weighted percent sand value for that subbasin. In any instances where weighted percent sand values greater than 100 percent were computed, a 100 percent sand value was adopted, to be consistent with the derivation of the urbanization curves.

As mentioned previously, the (NRCS *Soil Survey of Cooke County, Texas*, was consulted to determine specific soil types within the study area. The numerous soil types were grouped into four basic categories (0, 33, 67, and 100 *percent sand)* and weightings were applied within each runoff subbasin as shown in the following table. Net "percent sand" values were rounded to the nearest 5 percentage points for application in this study. Based on these investigations, this watershed would be expected to respond fairly similarly to a similar watershed in the Blackland Prairie Region.

Table 5 - Development of Percent Sand Values

Pecan Creek Subbasin Name	Estimated Percent which is 0 Percent Sand	Estimated Percent which is 33 Percent Sand	Estimated Percent which is 67 Percent Sand	Estimated Percent which is 100 Percent Sand	Weighted Percent Sand	Weighted Percent Sand Rounded to Nearest Five
P2	90	0	0	10	10	10
P3	80	0	0	20	20	20
P4	90	10	0	0	3	5
P5	85	0	0	15	15	15
P6	90	10	0	0	3	5
P7	75	15	0	10	15	15
P8	90	5	0	5	7	5
P9	80	0	0	20	20	20
P10	75	10	5	10	17	15
P11	85	10	0	5	8	10
P12	70	20	0	10	17	15
P13	100	0	0	0	0	0
P14	85	0	5	10	13	15
P15	75	5	10	10	18	20
P16	50	5	25	20	38	40
P17	90	10	0	0	3	5
P18	75	10	0	15	18	20
P19	75	0	10	15	22	20

Pecan Creek Subbasin Name	Estimated Percent which is 0 Percent Sand	Estimated Percent which is 33 Percent Sand	Estimated Percent which is 67 Percent Sand	Estimated Percent which is 100 Percent Sand	Weighted Percent Sand	Weighted Percent Sand Rounded to Nearest Five
P20	45	45	0	10	25	25
P21	45	10	35	10	37	35
P22	30	5	50	15	50	50
				minima:	0	0
				median:	17	15
				mean:	17	17
				maxima:	50	50

Determination of Rainfall Losses

Runoff volumes, excess rainfall amounts, were computed by deducting applicable losses from incremental rainfall amounts. Block, or initial abstractions and uniform infiltration rate losses were applied to all pervious soil surfaces within each subbasin. Standardized Fort Worth District US Army Corps of Engineers methodologies utilize the following array of initial abstractions and infiltration rates in combination with the *percent sand* values derived as described above to define rainfall losses within each runoff subbasin. These loss patterns are configured to consider the fact that greater antecedent conditions tend to prevail during periods of the year that are most susceptible to rare thunderstorm activity. These specific losses were developed from flood hydrograph reproduction studies on watersheds in the general Dallas-Fort Worth area and were adopted for this study area based on the similarity of soils and runoff characteristics.

Table 6 - Initial Loss Rates

		Sandy Soils		Clayey Soils	
Exceedance Probability	Return Interval (years)	Initial Abstraction (inches/hour)	Infiltration (inches/hour)	Initial Abstraction (inches/hour)	Infiltration (inches/hour)
50	2	2.1	0.26	1.50	0.20
20	5	1.8	0.21	1.30	0.16
10	10	1.5	0.18	1.12	0.14
4	25	1.3	0.15	0.95	0.12
2	50	1.1	0.13	0.84	0.10
1	100	0.9	0.10	0.75	0.07
0.2	500	0.6	0.08	0.50	0.05

These losses are based on an analysis originally done in 1957. In this analysis, the initial abstractions and infiltration rates were determined for 10 storm reproductions on the East Fork of the Trinity River near Rockwall, Texas. Losses from these storm reproductions ranged from maximums of 1.30-inch initial abstraction and 0.16-inch per hour infiltration, to minimums of 0.50-inch initial abstraction and 0.05-inch per hour infiltration. Based on these storm reproductions, the 2-year frequency storm was assigned an initial abstraction and infiltration rate of 1.50 inches and 0.20 inch per hour, respectively. The 1000-year frequency storm was assigned an initial abstraction and infiltration rate of 0.50 inches and 0.05 inch per hour, respectively. Losses for the 5-year through 100-year frequency storms were then interpolated. Later studies adopted the "1-year" losses to be the same as those for the 2-year event and the losses for the 500-year and SPF events to be the same as those for the 1000-year event. An additional 30 storm reproductions were used in the development of the Blackland Prairie Clay and Cross Timber Sandy Loam Urbanization Curves (shown above in Figures 2 and 3) in 1970 and 1977. In the analysis of these storm reproductions, it was determined that the losses calculated in 1957 more closely matched those for the watersheds that were predominantly clayey in nature; therefore, they became the "clay" losses. A companion set of "sand" losses were then developed by increasing the "clay" losses, using losses determined from storm reproductions in the sandy watersheds as a guide. Subsequent studies, including streamflow frequency analyses have been used to verify the reasonableness of these losses. They have been applied successfully in studies throughout the state, since they relate to soil type, rather than to a specific geographic region.

Determination of Snyder's Unit Hydrograph Lag Times

Each of these previous mentioned reports discuss the development of the Blackland Prairie Clay and Cross Timber Sandy Loam urbanization curves for the general Dallas-Fort Worth vicinity of Texas. These curves relate T_p to certain measurable subbasin parameters for a specific percent urbanization and soil type ("percent sand"). Each set of curves was based on flood hydrograph reproductions of predominantly clayey or sandy watersheds in the Dallas-Fort Worth area. The pertinent data for these flood hydrograph reproductions is presented in Tables 9 and 10. These curves have been successfully applied to a number of flood insurance and planning studies in Texas with satisfactory results. They are displayed on Figures 2 and 3. The urbanization curves relate T_p to the quantity:

$$(L)(L_{CA})/(S_{ST})^{0.5}$$

where:

(1) T_p -   the lag time (hours) from the midpoint of the unit rainfall duration to the peak of the unit hydrograph

(2) L -   the stream mileage from the subbasin outlet to the upstream limits of the subbasin

(3) L_{CA} -   the stream mileage from the subbasin outlet to the geographical centroid of the subbasin

(4) S_{ST} -   the weighted stream slope (feet per mile) over the specific stream reach from 10 percent of L to 85 percent of L, with both being measured from the subbasin outlet

Table 7 – Data Used in Developing the Blackland Prairie Clay Urbanization Curves

Gauge Location	DA (sq mi)	Average Rainfall (inches)	Direct Runoff (inches)	Observed Peak (cfs)	CP640	Snyder's Lag (hours)	Date	$(L)(L_{CA})/(S_{ST})^{0.5}$	Urban
White Rock Creek at	29.4	3.39	1.49	8,300	590	2.77	6-May-69	27.6	4
Keller Springs Road		1.97	0.84	4,420	612	3.5	29-30 Jun 62	27.6	0
		5.84	2.5	9,410	686	3.5	27-Jul-62	27.6	0
		1.77	0.77	3,460	423	2.5	27-Sep-64	27.6	0
		2.35	0.9	3,170	605	3.5	18-Nov-64	27.6	0
		1.75	0.65	4,560	868	3.5	27-28 May 64	27.6	0
		2.52	1.43	9,020	763	3.5	28-Apr-66	27.6	0
				Averages:	650	3.25			1
White Rock Creek at	66.4	3.93	1.65	24,500	620	2.5	8-Oct-62	78.4	10
Greenville Avenue		1.56	0.7	6,940	759	4.5	27-Sep-64	78.4	10
		1.96	1	7,500	626	4.5	18-Nov-64	78.4	10
		3.37	2.44	11,000	616	5.5	8-9 Feb 65	78.4	10
		2.64	1.77	13,800	469	3.5	10-11 May 65	78.4	10
				Averages:	618	4.1			10
Turtle Creek at	7.98	1.73	0.64	3,050	439	0.75	30-Apr-62	2.9	100
Dallas, Texas		4.36	2.18	4,640	338	0.75	27-Jul-62	2.9	100
		3.8	1.68	3,450	291	0.75	8-Oct-62	2.9	100
		2.82	1.81	4,290	479	0.75	28-Apr-63	2.9	100
		2.12	1.66	4,520	489	0.75	19-May-65	2.9	100
		3.55	3.04	12,200	658	0.75	28-Apr-66	2.9	100
				Averages:	449	0.75			100
Bachman Branch	10	5.35	3.21	9,200	255	0.75	8-Oct-62	2.6	
at Midway Road		4.1	1.28	3,620	468	1.25	20-21 Sep 64	2.6	
		1.1	0.69	2,910	337	0.75	21-Sep-64	2.6	
		1.36	0.58	3,050	195	0.25	22-Sep-64	2.6	
		2.36	1.52	5,170	595	1.25	10-May-65	2.6	
				Averages:	370	0.85		2.6	70
Joes Creek at	7.51	4.77	3.71	6,350	316	1.25	28-Apr-66	2.9	55
State Highway 114		4.64	3.28	7,300	380	0.58	8-Oct-62	2.9	50
				Averages:	348	0.92		2.9	53
Duck Creek at	31.6	4.38	3.52	10,500	550	3.83	6-May-69	23.5	37
Garland, Texas		3.56	1.99	7,400	388	2.5	26-Apr-58	23.5	37
(Beltline Road)		0.77	0.64	2,140	364	3.5	28-29 Apr 58	23.5	
		3.05	1.09	4,160	358	2.5	1-Oct-59	23.5	
		7	4.54	16,000	395	2.5	27-Jul-62	23.5	

Gauge Location	DA (sq mi)	Average Rainfall (inches)	Direct Runoff (inches)	Observed Peak (cfs)	CP640	Snyder's Lag (hours)	Date	$(L)(L_{CA})/(S_{ST})^{0.5}$	Urban
		3.02	1.63	7,400	530	2.5	28-Apr-63	23.5	
		3.53	1.78	5,620	497	3.5	9-Feb-65	23.5	
		3.91	2.72	9,500	325	2.5	28-Apr-66	23.5	
		2.46	1.81	8,600	395	2.5	29-Apr-66	23.5	
				Averages:	422	2.87		23.5	35
Big Fossil Creek	53.8	6.15	3.73	27,200	590	3.63	7-Sep-62	47.6	5
at Haltom City		4.43	2.69	7,770	420	4.5	25-26 Apr 57	47.6	5
		5.86	4.62	13,000E	420	4.5	25-26 May 57	47.6	5
		6.54	2.06	18,300	603	3.5	24-25 Jun 61	47.6	5
		5.29	1.69	12,600	462	3.5	30 Sep-1 Oct 59	47.6	5
				Averages:	499	3.93		47.6	5
Village Creek at	130	1.54	0.5	4,180	460	5.2	19-May-26	95.6	
Handley, Texas		3.41	0.85	9,400	460	5.2	1-Oct-27	95.6	
		3.46	1.38	14,800	460	5.2	17-Dec-28	95.6	
				Averages:	460	5.2		95.6	0
Duck Creek at	7.9	2.5	0.9	2,500	250	0.64	30-May-70	5.32	40
Buckingham Road		3.17	1.79	3,960	280	0.89	16-Sep-74	5.32	60
				Averages:	265	0.77		5.32	50
South Mesquite Creek	13.4	2.89	1.86	3,420	400	1.89	23-Apr-73	6.61	65
at Highway 352		2.3	1.58	3,090	420	1.75	20-Sep-73	6.61	65
				Averages:	410	1.82		6.61	65
Marine Creek at	17.3	1.45	0.56	1,680	405	2.25	26-Apr-57	6.3	5
NW 33rd Street									
Sycamore Creek	17.7	1.38	0.28	1,140	450	1.77	30-May-70	7.19	18
at I.H. 35W									
Rowlett Creek near	120.3	3.4	2.01	24,400	600	8.36	6-May-69	112.7	4
Sachse (Highway 78)		3.41	2.48	24,700	600	6.62	9-Dec-71	112.7	6
				Averages:	600	7.49			5
Five Mile Creek at	37.9	1.85	0.7	7,040	460	1.55	30-May-70	8.44	55
Lancaster, Texas		2.36	0.85	8,540	440	1.55	7-Jul-73	8.44	55
				Averages:	450	1.55			55
Five Mile Creek	13.2	3.02	1.45	6,180	300	0.49	30-May-70	5.91	55
at Highway 77									
Ten Mile Creek	52.8	3.74	1.86	8,820	320	2.91	3-Jun-73	24.77	25
at Highway 342									
Cedar Creek at	9.42	2.15	0.75	4,840	350	0.45	29-May-73	2.42	100
Bonnie View Road									
Coombs Creek at	4.75	4.4	2.73	2,960	320	0.82	6-May-69	1.64	100

Gauge Location	DA (sq mi)	Average Rainfall (inches)	Direct Runoff (inches)	Observed Peak (cfs)	CP640	Snyder's Lag (hours)	Date	$(L)(L_{CA})/(S_{ST})^{0.5}$	Urban
Sylvan Avenue									
Little Fossil Creek	12.3	2.12	1.28	1,530	260	2.27	6-May-69	7.9	18
at Mesquite Street		1.61	0.83	1,370	280	1.91	30 Apr-1 May 70	7.9	18
				Averages:	270	2.09		7.9	18
Mountain Creek	103.5	3.38	2.35	18,500	433	5.6	25-26 Apr 70	86	1
near Cedar Hill		5.92	4.81	28,300	324	4.95	6-8 May 69	86	1
				Averages:	379	5.28		86	1
Honey Creek SCS Site	1.26	1.46	1.46	1,170	360	0.4	29-Apr-58	0.216	2
No. 12 near McKinney		1.62	1.62	1,480	380	0.26	1-May-58	0.216	2
		1.03	0.95	850	550	0.73	28-Apr-66	0.216	2
		1.72	1.72	1,400	550	0.62	30-Apr-66	0.216	2
				Averages:	460	0.5		0.216	2
Honey Creek SCS Site	2.14	1.22	0.93	1,250	340	0.5	28-Apr-66	0.177	2
No. 11 near McKinney		2.36	2.3	3,230	320	0.33	30-Apr-66	0.177	2
				Averages:	330	0.42		0.177	2

Table 8 - Data Used in Developing the Cross Timber Sandy Loam Urbanization Curves

Gauge Location	DA (sq mi)	Average Raingall (inches)	Direct Runoff (inches)	Observed Peak (cfs)	CP640	Snyder's Lag (hrs)	Date	$(L)(L_{CA})/(S_{ST})^{0.5}$	% Urban
Walnut Creek (90% sandy)	62.8	3.22	1.45	4,730	450	7.87	8-10 Feb 65	46.5	1
near Mansfield (10% clay)		2.08	1.22	5,390	550	7.87	30 Apr-1 May 66	46.5	1
		4.7	1.67	6,840	550	7.87	6-7 May 69	46.5	1
Rush Creek at Arkansas Ln	27.11	2.89	1.2	2,500	460	5	12-13 Oct 73	17.2	5
Big Sandy Creek (40 60% sandy)	333	5	3.15	53,000	640	13.4	10-12 Jun 41	564	1
Near Bridgeport (40% clay)	333	4.4	1.98	17,350	610	21	7-10 Apr 42	564	1
				Averages	625	17.2			
Big Sandy Creek (80% sand)	233.2	8.72	2.86	19,110	400	11.3	3-6 Oct 59	202	1
Near Bridgeport (20% clay)									
(Area modified by Lake Amon Carter)									

The degree of urbanization (percent urbanization) for use with these methodologies was estimated as the percentage of each subbasin that has been improved with channelization and storm drainage facilities, including the existence of street "curb and gutter" networks.

This parameter is meant to focus solely upon the characteristics of the subbasins that would tend to increase the speed at which excess runoff is conveyed to the primary streams. Within the mathematical model itself, this parameter affects only the unit hydrograph lagging time. Methods used to make these estimates include examination of USGS 7.5-minute and detailed topographic maps, aerial photos, city land use maps, and via field reconnaissance trips, etc.

Imperviousness ("percent impervious") values for each subbasin were estimated using a similar methodology, except that since this parameter is meant to directly identify the portion of each drainage area which is incapable of infiltrating surface floodwaters, it requires significantly less engineering judgment than that required for estimation of the percent urbanization parameter.

Table 9 presents unit hydrograph data for each subbasin as identified on the previously mentioned Figure 1.

Table 9 - PECAN CREEK RUNOFF SUBBASIN DATA

SUBBASIN NAME	AREA (sq.miles)	LENGTH (miles)	LENGTH TO CENTROID (miles)	WEIGHTED STREAM SLOPE (ft./mile)	PERCENT SAND	PERCENT URBAN.	PERCENT IMPERV.	SNYDER'S UNIT HYDROGRAPH LAG TIME (hours)
P17	1.40	2.41	1.12	29	5	10	5	0.69
P12	0.79	2.02	0.90	26	15	0	0	0.69
P13	0.78	2.23	0.99	41	0	10	5	0.58
P11	0.37	1.91	1.08	35	10	10	5	0.62
P22	1.44	2.71	1.29	49	50	0	0	0.99
P21	0.79	2.16	1.23	52	35	0	0	0.80
P19	1.43	3.85	1.93	18	20	0	0	1.31
P20	0.86	2.13	1.14	41	25	0	0	0.75
P18	1.38	2.69	1.50	29	20	10	5	0.89
P14	1.08	2.66	1.67	17	15	0	0	1.05
P9	0.28	1.23	0.54	36	20	10	5	0.43
P16	0.89	2.47	1.35	45	40	0	0	0.92
P15	0.35	1.79	0.99	45	20	0	0	0.63
P10	0.52	2.55	1.34	19	15	0	0	0.93
P8	0.76	2.19	0.98	35	5	40	20	0.51
P6	0.37	1.06	0.40	38	5	60	30	0.24
P7	0.49	1.42	0.65	51	15	70	35	0.30
P5	0.27	1.11	0.35	46	15	90	45	0.20
P4	0.25	1.16	0.44	28	5	80	40	0.24
P3	0.59	1.64	0.69	31	20	70	35	0.37
P2	0.37	1.74	1.25	28	10	20	10	0.62

Routing of the flood hydrographs through each stream reach was accomplished via the standardized modified Puls methodology.   Discharge versus storage relationships was developed from outputs of concurrently developed backwater modeling for this study.

Distribution and transposition of the hypothetical storm data, subtraction of initial abstractions and hourly infiltration losses, computation of Snyder's unit hydrographs and actual flood hydrographs, and summarization of flood hydrograph analysis results were all accomplished via the US Army Corps of Engineers' "HEC-1" Flood Hydrograph (software) Package.   Existing conditions computed probability discharge versus frequency relationships for Pecan Creek are shown in Table 10.

Table 10 - SUMMARY OF EXISTING CONDITIONS PEAK DISCHARGES (cfs)

LOCATION DESCRIPTION	CONTRIBUTING WATERSHED AREA	ANNUAL EXCEEDANCE PROBABILITY (percent)							
		50	20	10	4	2	1	0.4	0.2
		RECURRENCE INTERVAL (years)							
	(sq.miles)	2	5	10	25	50	100	250	500
Headwaters	1.40	1184	1941	2293	2733	3095	3450	3967	4327
Above IH-35 Trib	2.19	872	1550	1920	2335	2679	3025	3491	3830
At IH-35	2.97	1510	2572	3123	3771	4302	4836	5574	6102
Above North Fork (Pec-3)	3.34	1056	2017	2639	3317	3876	4420	5137	5662
Below North Fork (Pec-3)	10.32	1908	3781	4999	6317	7498	8695	10199	11354
Above Northeast Fork (Pec-2)	10.60	1638	3463	4649	5884	6989	8264	9778	11000
Below Northeast Fork (Pec-2)	12.36	1957	4212	5672	7155	8457	9933	11707	13199
Below Cloud Street Drain	13.12	1857	4102	5624	7184	8647	10194	12055	13522
Above Subbasin P6 Drain	13.61	1722	3844	5343	7003	8532	10117	12019	13485
Below Subbasin P6 Drain	13.98	1728	3860	5370	7049	8597	10202	12126	13602
Above Subbasin P4 Drain (California St)	14.25	1724	3621	4985	6578	8018	9624	11441	12971
Below Subbasin P4 Drain (California St)	14.50	1730	3629	4999	6602	8052	9669	11496	13035
At Anthony Street	15.09	1694	3492	4810	6349	7702	9280	11185	12722
Above Wheeler Creek	15.46	1597	3304	4547	6118	7382	8801	10634	12201

Hydraulics Background

In May 1999, HDR Engineering, Inc completed a *Flood Protection Planning Study* for the City of Gainesville, Texas under a grant from the Texas Water Development Board. As part of this study, HDR developed a "HEC-RAS" backwater model for Pecan Creek from its confluence with the Elm Fork of the Trinity River to 2.4 miles upstream of the IH-35 crossing, northwest of the city. Their hydraulic model was used as a baseline for the current study. The various input parameters were either verified or updated, as necessary for compatibility with standard modeling methodologies used on preceding Section 205 Flood Damage Reduction Feasibility Studies by the Fort Worth District, US Army Corps of Engineers. During this process, most of the modifications to HDR's backwater model related to Manning's roughness coefficients and bridge/culvert parameters, including the manner with which ineffective flow areas were defined adjacent to the occasional obstructions to flow.

Both without- and a series of with-project conditions water surface profiles were computed for Pecan Creek for the study reach defined above. Steady state flow conditions were analyzed for the 50-, 20-, 10-, 4-, 2-, 1-, 0.4- and 0.2-percent annual chance exceedance (2-, 5-, 10-, 25-, 50-, 100-, 250-, and 500-year) flood events. Manning's roughness coefficients were determined from aerial photographs and field investigations. Computed water surface profiles for existing and with-project conditions are shown on Figures 4 through 6, shown at the end of this Appendix. Figure 4 shows the existing 2-, 10-, and 100-year computed water surface elevations. Figure 5 shows the with-project 2-, 10-, and 100-year computed water surface elevations. And Figure 6 shows a comparison between the existing and with-project 100-year computed water surface elevations.

Existing Conditions

Existing conditions were modeled from the mouth of Pecan Creek to 2.4 miles upstream of the I-35 crossing of Pecan Creek. The reach downstream from Anthony Street has not been significantly modified. In contrast, the reach between Anthony and Scott Streets has had its channel reconfigured into a trapezoidal shape, with an approximate bottom width of 15 feet with 1 on 1.5 side slopes. As these modifications were being implemented, the channel was also straightened considerably, especially along the reach between Anthony and Garnett Streets. Furthermore, within this reach between Garnett and Scott Streets, the lower four feet of the channel banks and its bed are lined with flagstone and concrete. Upstream from Scott Street, the channel is generally trapezoidal shaped and unlined. A cross section layout map is provided as Figure 7, shown at the end of this Appendix.

Alternatives Analysis

In order to determine what types of flood damage reduction solutions merited consideration along Pecan Creek, the existing (without-project) analysis results were compared with a concurrently-developed flood damage economics dataset. This comparison revealed the existence of a major damage center in the reach between Moss and Broadway Streets, encompassing the heavily-developed area adjacent to downtown Gainesville. Another major damage center, composed primarily of single-family residential properties, was noted in the reach extending upstream from Broadway Street, past Belcher Street.

Downstream from Moss Street, potential damage centers appeared to be limited to a small group of mobile homes situated in the right (west) overbank of Pecan Creek, between Anthony and Moss Streets.

Initially, a broad array of potential alternatives were brainstormed and given at least cursory consideration. These included upstream detention and floodplain evacuation (relocation). In each case either the projected implementation costs, when annualized, exceeded potential expected annual benefits or separate constraints limited the feasibility of those proposals. As the study progressed, channel modification alternatives continued to consistently show the greatest promise for providing the necessary Federal interest.

Since channel improvements in downstream reaches would tend to provide some incidental reductions in flood stages in adjoining, upstream reaches, the decision was made to investigate channel modifications, beginning at the most downstream damage center. It was anticipated that economically-optimized solutions could be configured in series, such that the overall product would be coincidentally optimized.

Through a series of trials, it was determined that the relatively minor amount of expected annual damages associated with the reaches downstream from Moss Street would not provide sufficient benefits to economically justify any major channel modifications in those reaches.

It was noted that the next upstream, significant damage reach, was situated far enough upstream (about 1,500 feet) from the Moss Street bridge, that modifications to that structure would have only minor impacts upon flood stages in the damage center itself. Therefore, the next potential channel modification reach given intensive consideration was that extending from about halfway between Moss and Garnett Streets, to Broadway Street, which defined the upstream boundary of the most intensively developed commercial property zone along Pecan Creek.

In general, most of the channel improvement scenarios considered were for a straightforward, grass-lined, earthen, trapezoidal channel section, with 3.5:1 side slopes. This side slope was selected to assist the City in maintenance of the project since steeper slopes can present a hazard for mowing. Since the anticipated channel bank height was known to be on the order of 10 feet, it was recognized that this approach would entail relatively wide project rights-of-way, amounting to 70 feet plus whatever bottom width was being considered. Therefore, specifically for this near-downtown reach, consideration was also given for more steeply-sided templates, such as concrete- and/or gabion-lined channel modifications. Of those narrower solutions, it was anticipated that the gabions scenario would have better chances for economic feasibility, while simultaneously providing a more aesthetically pleasing project.

A detailed hydraulic analyses was performed for three gabion-lined and three grass-lined channel plans for the reach extending from upstream of Moss Street to just upstream of Broadway Street. The gabion-lined alternative reach was modeled with 1.5:1 side slopes and the grass-lined alternative reach was modeled with 3.5:1 side slopes. The channel's

bottom width was used as the primary variable while performing plan formulation, comparitive analyses.

The channel invert profile was defined by matching the existing grade at the downstream end of the modified reach, which was just upstream of Moss Street, and transitioning to a reasonable invert elevation at the Broadway Street bridge, based on relatively minor invert excavation at that upstream interface point. As the with-project backwater analyses were being developed, it became clear that the existing stream invert slope would produce erosive velocities in a modified channel with Manning's roughness on the order of that for grass-lining, not to mention even less resisitant boundary materials. As a result, the decision was made to flatten the channel profile slightly, to a value of 0.25 percent. This meant that the proposed invert elevation at the Broadway Street bridge would be about three feet below the existing grade at that point.

As mentioned above, the gabion-lined alternative was considered primarily as a means of providing a significant increase in channel conveyance while minimally increasing the required right-of-way acreages. In configuring this alternative, however, only the intensely encroached reach between Garnett and Broadway Streets was targetted for gabion-lining. The remaining downstream portion of the improved reach, between Garnett Street and Moss Street, was configured for the wider, grass-lined channel cross section.

Gabion-lined channel bottom widths of 20-, 45-, and 70-feet and grass-lined bottom widths of 30-, 65-, and 100-feet were analyzed. In setting the range for the bottom width of the proposed grass-lined channel cross section, 30-feet was generally considered the minimum, so as to avoid filling any of the existing channel bottom, and 100-feet was considered adequate to fully convey the 100-year flood discharge within the channel area. Conveyance-equivalent bottom widths for the gabion-lined channel cross section template were 20- and 70-feet, respectively. As these initial plan formulation scenarios were being finalized, comparisons of annualized project costs with expected annual benefits indicated that the additional costs for the gabion lining in the intensively developed middle reach far outweighed the potential savings in real estate costs. Therefore, only the grass-lined channel alternatives were investigated in further detail during this feasibility study.

Once sufficient economic evaluations had been performed to confirm National Economic Development (NED) optimization in the Moss Street - to - Broadway Street reach, the proposed project reach was then extended upstream, about 1,000 feet upstream of Belcher Street, using the same 0.25 percent longitudinal invert slope. The channel invert would be deepened by a maximum of 4.9 feet and by an average of 3.3 feet over the modified reach. The centerline of the modified channel was positioned so as to minimize impacts to existing structures and the railroad embankment. The proposed modified channel length is approximately 8,000 feet (1.52 miles). It includes compatible modification of six bridge structures.

As the plan formulation phase was nearing completion, it was noted that the "100-foot bottom width" plan was providing for little, if any, flood damage reduction benefits above and beyond those afforded by the "65-foot bottom width" plan. In addition, net excess

benefits of the "30- and 65-foot bottom width" plans were nearly equal, implying that a truly economically optimized solution might involve a project within that lower range of sizes. The decision was made to insert an additional "50-foot bottom width" plan into the plan formulation array, as the likely Selected Plan. Subsequently, however, as project benefits and costs were being further scrutinized and finalized, it was determined that the net excess benefits for the narrower, "30-foot bottom width" plan actually exceeded those for the "50-foot bottom width" plan, thereby supplanting it as the officially Selected Plan.

Following are tabulated summaries of reductions in computed 100-year water surface elevations and anticipated flow velocities, for selected plan.

Table 11 – Summary of Reduction in Computed Water Surface Elevations Between Alternatives

Reduction in CWSEL	30-foot BW Plan	50-foot BW Plan	65-foot BW Plan
Maximum (feet)	4.8	7.4	8.4
Average (feet)	2.9	5.1	6.1

Table 12 – Comparison of Computed Channel Velocities between Existing Conditions and the Selected Plan

	Minimum		Maximum		Average	
Frequency	Existing	Modified	Existing	Modified	Existing	Modified
2-year	1.57	2.19	13.46	11.19	6.00	4.48
10-year	1.99	1.87	13.59	8.44	5.89	5.14
100-year	2.49	2.37	13.84	10.89	6.14	6.45

Prior to finalization of the feasibility phase of study, specific analyses were performed to determine the significance of any downstream impacts which could occur as a result of the reductions in valley storage in the reach of proposed channel modification. With-project valley storage relationships were developed and applied within the watershed runoff model, in order to determine the potential increases in peak discharges, downstream of the proposed project. The hydrologic analysis node at Anthony Street, provides the peak discharge array applied along the potentially-impacted reach within the City of Gainesville. At this node, the proposed project has the potential to increase hypothetical flood peak discharges by 1 to 12 percent, depending upon the flood frequency of interest. The impact would be greatest for events on the order of those having a 5- to 10-year recurrence interval. At the 100- and 500-year events, the impact upon peak discharges would fall to 5 and 1 percent, respectively.

For purposes of determining potential economic impacts (i.e. project disbenefits), the with-project exceedance probability function was applied within the HEC-FDA model. Those results are presented in the main report. For purposes of determining specific impacts upon computed water surface elevations, the with-project hydrologic analysis results were applied within the backwater model. Increases in flooding depths would average about 0.15 feet (1.8 inches) over the reach in question, within the City of Gainesville. The most significant average impact, 0.26 feet (3.1 inches) would occur at the 10-year recurrence

interval event. At the 100- and 500-year events, the average impact falls to 0.13 feet (1.6 inches) and 0.04 feet (0.5 inches), respectively. The maximum impact upon the 100-year flooding depth would be 0.14 inches (1.7 inches).

Additional investigations revealed that these inducements upon the water surface elevations could be generally mitigated, if the local sponsor would implement relatively minor channel and overbank maintenance over the reach between the downstream end of the proposed channel modification and FM 2071 (Old Denton Road).